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By Seismic Research Group, Architectural Institute of Japan; Reuben W. Binder; R. R. Martel, G. W. Housner, and J. L. Alford; S. K. Guha and Gurdas Ram; and a Joint Committee of the San Francisco, California Section, ASCE, and the Structural Engineers Association of Northern California

STRUCTURAL DIVISION

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DISCUSSION

SEISMIC RESEARCH GROUP, ARCHITECTURAL INSTITUTE OF JAPAN.—Although Japan has experienced frequent violent earthquakes in the past, no seismograms useful to engineers have been obtained, except at the EARTHQUAKE RESEARCH INSTITUTE OF TOKYO UNIVERSITY, TOKYO, JAPAN, during the 1923 KANTO EARTHQUAKE. Japanese engineers have been unable to obtain the necessary support to establish and maintain strong-motion seismographs at the various meteorological stations throughout Japan.

The proposal of the authors has shown the way for theoretical determination of the design coefficient by fully utilizing strong-motion seismograms. It is a remarkable achievement and deserves the highest praise. The SEISMIC RESEARCH GROUP wishes to congratulate the authors for their contribution and to express admiration for their development of the dynamic method of determining lateral force.

Although there are no reliable strong-motion seismograms to use for definite comparison, it is hoped that what data have been accumulated, together with Japanese experience, will make this discussion useful.

Effect of Soil Type.—The condition of the soil is considered to be the most influential factor in the consideration of seismic force upon a building. A building founded on hard rock normally suffers little damage unless it is of very fragile construction. For hard rock, the seismic design coefficient can be very small. As the soil becomes softer the possibility of damage increases and differences that result from the use of differing construction materials become visible. The design coefficient may be varied because of varying conditions of the nature of the site soil. K. Muto (see "Acknowledgment") has proposed reduction percentages to be applied to the design coefficient for the various values of soil bearing capacity. Table 5 shows the proposed reduction values which soon will be adopted into the Japanese National Building Code.

TABLE 5.—PERCENTAGE EFFECT OF SOIL TYPE ON SEISMIC FORCE

Soil	Type of Construction			Corresponding soil bearing power (kips per sq ft)
	Wood and light steel	Reinforced concrete and concrete block	Masonry	
Soft	100	100	100	1.0 or less
Intermediate	80	90	100	....
Hard	60	80	100	4.0 or more

According to surveys made by R. Tanahashi and others on damages caused by past violent earthquakes, ordinary wooden buildings with heavy roof framing members and tile are certain to collapse when built upon soft soil.

NOTE.—This paper by a Joint Committee of the San Francisco, California Section, ASCE, and the Structural Engineers Association of Northern California was published in April, 1951, as *Proceedings-Separate No. 66*. The numbering of footnotes, illustrations, tables, equations, and Bibliography are a continuation of the consecutive numbering used in the original paper.

This is true unless such structures are designed for an ultimate design coefficient not less than 0.3 g to 0.4 g. Within the area of violent earthquake motions, the observed damage rate to wooden structures decreases quickly when the structure has been built on intermediate soils. On hard soils the observed damage rate to wooden buildings is usually very small and of a minor nature. The average natural period of these wooden buildings is around 0.5 sec within the small vibrational period range.

The damage surveys on reinforced concrete buildings, either with or without structural steel composite reinforcing, after the Kanto Earthquake of September, 1923, reveal that the over-all damage rate (including all minor cracking) was least for buildings situated on the soft soil in the downtown (Marunouchi) area of Tokyo. However, recent extensive surveys conducted by T. Kitazawa on all damage reports submitted for this earthquake shows that, when the damage rate is compared on the basis of damage to main structural members, the buildings located upon the downtown soft soil areas suffered considerably more than did those located on the uptown (Hongo) intermediate soil areas of Tokyo. Hard and intermediate soils are much more preferred for this type of construction than is soft soil.

TABLE 6.—Period Of Buildings In Tokyo, Japan

Building	Height, in feet	East–West Direction				North–South Direction			
		Width b, in feet	Period, in Seconds			Width b, in feet	Period, in Seconds		
			Actual, T_A		Computed[b]		Actual, T_A		Computed[b]
			Normal	1923[a]			Normal	1923[a]	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Marunouchi . . .	110	280	0.67	1.11[c]	0.33	330	0.71	1.18[c]	0.30
N Y K	100	160	0.69	0.90	0.40	300	0.77	0.80	0.30
Tokyo Kaikan. .	86	144	0.72	1.30	0.36	144	0.54	1.20	0.36
Nippon Kogyo.	99	155	0.65	. . . .	0.40	131	0.61	. . . .	0.43

[a] Period after the September, 1923, earthquake. [b] Computed values of T_e using Eq. 9b. [c] Prior to the earthquake of April, 1922, the periods were 0.87 east–west and 0.94 north–south.

Small earthquakes produce more damage on stone or masonry structures on hard soil than on soft soil. However, violent earthquakes usually inflict severe damage on this type of structure even though it is built on soft soil. There are some notable exceptions to this general observation but generally the softer soils are the safest for this type of construction.

Effect of Period.—In Japan, buildings with a natural period of about 1 sec are considered as having a critical period when built upon soft soils. The damage to buildings in the downtown soft soil area of Tokyo in the 1923 Kanto Earthquake was greatest to those buildings having a natural period of about 1 sec. Table 6 has been compiled to give comparative information on the measured natural periods of several buildings in this soft soil area with the period value calculated using the authors' proposed period formula. It was

found, for buildings in an undamaged state, that the period determined by use of the formula is much shorter than the measured values.

Vibration tests on full-size and half-size test structures have confirmed conclusively the fact, already observed previously, that a structure's natural period of vibration changes in an earthquake. The natural period, even in an undamaged structure, elongates when subject to the larger amplitudes and accelerations of an earthquake. The natural period of a structure is, therefore, a transient value.

The Seismic Research Group, working in conjunction with the Building Research Institute of the Construction Ministry and the Earthquake Research

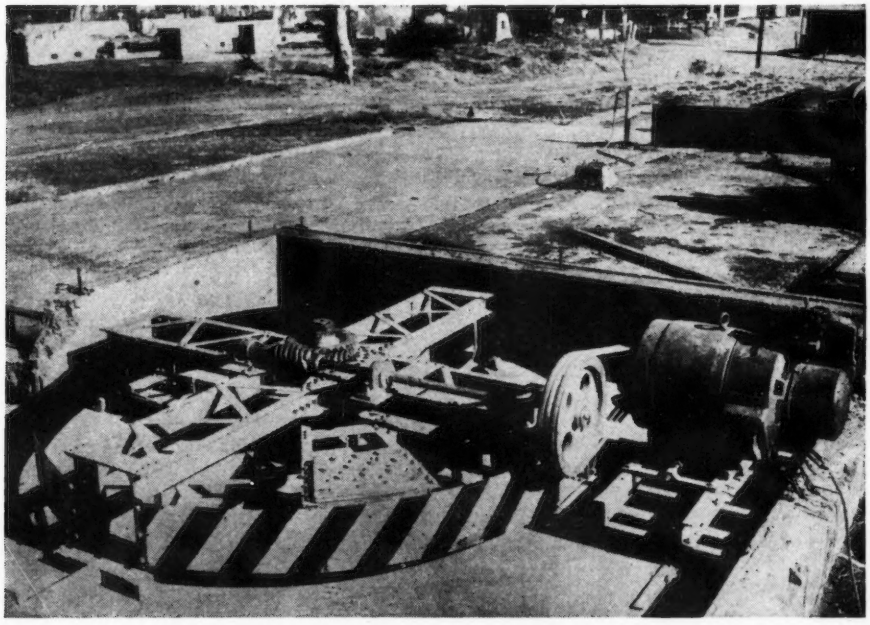

FIG. 13.—LARGE VIBRATION GENERATOR

Institute, has been conducting statical and vibrational tests on specially built full-size and half-size test structures. The Committee used a specially constructed large vibration generator (Fig. 13) on six types of structures and succeeded in attaining accelerations as large as 0.4 g to 0.7 g at the roof slab levels of these structures.

A portion of the test data is given in Table 7. Initial vibration tests were made with a vibrator that had the following characteristics: 4 vertical eccentric disks were used having a total weight of 46 kg, maximum eccentric distance of 11 cm powered by a 2-hp electric motor and exerting a maximum force of 4.7 tons at 15 rps. The final vibration tests were made with the larger vibrator that had the following characteristics: one 380 kg weight was used on a horizontal rotating arm 90 cm in length powered by a 10-hp electric motor

and exerting a maximum force of 10 tons at 5 rps (overloaded 100% during final tests).

The final observed periods using the large vibrator are compared in Table 7 with the initial periods that were observed using the small vibrator. The periods of vibration in two mutually perpendicular directions and corresponding accelerations measured at the top and bottom of the buildings are tabulated. The soil in these tests was Tokyo loam (red clay). This soil is in the intermediate class, having a design bearing capacity of between 3 and 4 kips per sq ft.

The observed periods in the final tests showed the following elongation percentages over the observed periods in the initial tests:

Type of construction	Percentage period elongation
Concrete block	140 to 230
Brick masonry	180
Reinforced concrete	200
Prefabricated reinforced concrete	300

All the buildings in the test program had a fairly large height-to-width ratio, and the tests on each structure were repeated several times. This accounts for predominant soil effects, especially tilting. However, it must be remembered that the soil was good intermediate soil.

TABLE 7.—RESULTS

Type of construction	Size Of Building Initial						
	Dimensions, feet	Height, feet	Stories	Period, seconds		Acceleration	
				N–S	E–W	Ground line	
						N–S	E–W
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Japanese style concrete block[a]	13 by 20	29	3	0.238	0.255	0.011	0.013
Japanese style concrete block[b]	12 by 18	18	2	0.185	0.159	0.027	0.035
American standard concrete block[c]	11 by 14	19	2	0.174	0.163	0.023	0.027
Medium-burned clay brick[d]	14 by 15	16	2	0.196	0.193	0.036	0.030
Reinforced concrete	11 by 21	15	3	0.122	0.130	....	....
Prefabricated reinforced concrete	13 by 13	20	2	0.205	0.256	....	....

[a] Hollow concrete block 18 in. × 19 in. × 6.7 or 8 in., using vertical and horizontal reinforcement. [b] Channel block 16 in. × 8 in. × 8 in. using vertical reinforcement. [d] Brick 8 in. × 4 in. × 2 in. without reinforcement.

The peak period of the frequency spectrum in the paper appears to be caused by the short period element of the ground motion. This coincides with the period that most Japanese seismologists recognize for intermediate soil areas. When the natural vibration period of the surface soil strata is excited, the value attained depends on the nature of the soil. The prevailing primary

period of the soft soil in the downtown Tokyo area is 0.6 sec, and the secondary period is 0.3 sec.

After the 1923 Kanto Earthquake, the Imperial Damage Committee headed by R. Sano debated earnestly and at length as to the advisability of using the prevailing periods as the basis for the design of structures in this area. If the prevailing period plays an important part in contributing to damage at the resonant stage, it was then argued that the buildings in this area were subjected to the most unfavorable conditions. However, numerous important buildings with natural periods very close to the primary period of this area were not damaged in the least. One of these was the Nippon Kogyo Bank building, designed by T. Naito, that stands as a contradictory manifestation to any such theory. Other factors against the use of prevailing periods were the knowledge that the natural period of the soil may elongate and vary during violent earthquakes and the known difficulty of transmission of free vibration periods from the soft soil to the building. The adoption of period consideration into practical design was, therefore, abandoned. There must be some definite law, however, that governs all cases concerning hard soils.

Acceleration Spectrum.—The standard acceleration spectrum proposed by the authors in the section on "Earthquake Forces" has been commented on by interested engineers in Japan. Mr. Sano states that the period of maximum

OF VIBRATION TESTS

TEST RESULTS		FINAL TEST RESULTS						
(Ratio to g)		Period, seconds		Acceleration (Ratio to g)				Wall damages
Roof				Ground line		Roof		
N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	
(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
0.039	0.046	0.325	0.348	0.171	0.110	0.610	0.404	Hair cracks
0.150	0.133	0.476	0.338	0.121	0.161	0.717	0.598	Moderate cracks
0.115	0.113	0.377	....	0.206	0.154	0.608	0.392	Moderate cracks
0.121	0.099	....	0.355	0.164	0.131	0.439	0.595	Large cracks
....	....	....	0.267	0.104	0.245	0.180	0.725	No damage
0.067	0.100	....	0.690	0.100	0.035	0.343	0.552	Failure

type concrete block 36 in. × 36 in. × 3 or 6 in., using vertical and horizontal reinforcement. [c] Hollow concrete Wall 8 in. thick. Reinforced concrete floors.

ground acceleration chosen (0.25 sec) seems to be too small based on past Japanese earthquake experience. Accounting for all soil factors, he believes a period of about 0.50 sec is more reasonable as an over-all period. S. Ban believes that a large acceleration with a larger period is especially critical but is not represented by the proposed spectrum. T. Taniguchi feels that the

spectrum represents conditions considered proper for intermediate soils in the near epicentral area of an intermediate earthquake. The peak would be expected to shift to the right for soft soils even comparatively remote from the epicentral area of a large intensity earthquake. Although Biot's spectrum for minor earthquake response had its peak at 0.2 sec, K. Kanai notes that Robison's spectrum for the El Centro Earthquake has its peak at 0.25 sec. If the El Centro seismogram is analyzed by Mr. Ishimoto's method, the predominant period is found to be exactly 0.25 sec. Judging from Japanese experience the predominant period becomes longer the more the earthquake intensity increases and the peak period can be expected to shift to the right to a larger value during high intensity earthquakes. Mr. Naito comments that the El Centro seismogram plays the key role in preparing this spectrum and influencing the authors' proposal. The El Centro record can be typical only if all earthquakes and all soil conditions are similar, which cannot be expected.

Mr. Muto states that the spectrum (Fig. 5) applies to perfectly elastic structures. It is necessary, therefore, before application for design use to make an adequate correction to the curve for damping. This would decrease and round out the peak values of the curve, decrease the rate of reduction, and increase the values for the larger periods. Messrs. Naito and Taniguchi have commented that the proposal states that it is sufficient to design for the fundamental mode only. It is very probable that for many of the tall, slender buildings in the United States, the second mode of vibration is the most effective and the additive effect of the first and second modes should be considered.

Based on the concept that at least the second mode of vibration should be considered, Mr. Muto and H. Umemura made calculations on two-story, five-story, and infinite-storied buildings based on the proposed spectrum. The results show that, in the case of the combined first and second modes, approximately 10% excess of the total base shear may occur. This seems to be well within the allowable excess for practical design.

Lateral Force Coefficients.—The value of the proposed lateral force coefficient C has also aroused Japanese comment. Past damage surveys and records indicate that on low, simple buildings force coefficients of not less than 0.2 were experienced in moderately severe earthquakes. Almost universal agreement was reached on the necessity for the reduction of the seismic force for tall buildings.

Mr. Sano stated that, although there may be some dispute for over-all areas, such values as proposed by the authors seem to be acceptable for the western United States. However, he does not believe that the formula is readily applicable to all buildings of intermediate period, say, 0.25 to 0.75 sec. The formula merely defines a change between maximum and minimum values. It would not make an appreciable difference even if the formula defined a linear or hyperbolic change. Messrs. Ban and Taniguchi comment that it is questionable whether the selection of 0.02 as the minimum value of C without regard for soil conditions was made on a sound basis. The choice seems to have been based on the fact that many buildings with less calculated resistance than this were undamaged in past major earthquakes. What were the measured resistances of these same buildings in a major earthquake? On any building the

only sure way of finding the actual resistance is by measurement during an earthquake. Mr. Kanai believes that the strong-motion seismograms should be basis enough for determining building response and that they represent a more exact approach than the use of engineering judgment only. Y. Otsuki feels that use of the vibrational period of a building to determine its force coefficient without considering the damping nature of the particular building makes for an incomplete cycle. Low, flexible buildings, in which the danger of phase exists, the possibility of reversal with an increase in the rate of deformation should receive careful consideration.

Messrs. Tanahashi and Ishizaki note that the magnitude of seismic force is not always proportional to the mass but rather theoretically proportional to the square root of the mass. Statistical studies of damage data prove this statement. Accordingly, if an appropriate design seismic coefficient is determined for heavy reinforced construction, it may be unsafe for lighter structures having the same natural period of vibration.

Distribution of Base Shear.—The proposed distribution of base shear has evoked comment in Japan, some of which is reported here. General agreement has been shown with the proposed triangular distribution. Some disagreement occurs in regard to the application to tall slender buildings, and Mr. Sano feels that the adoption of triangular distribution of the total shear may be excessively on the safe side. Mr. Naito notes that, when the building has basement floors, the nature of the soil may affect the vibrational characteristics of the building; and the distribution of the base shear may become different from those buildings without basements. However, Mr. Ban agrees with the proposed method of shear distribution over the building. Mr. Otsuki holds that the actual ground motion includes components of all periods. Every component has a definite number of frequencies; hence even an undamped system would never come to resonance with any of these components. This is the meaning of the frequency spectrum. However, every structure is loaded critically only when it is shaken by the component of the resonant period. As far as this concept holds, the proposed triangular distribution works satisfactorily.

Mr. Muto and Mr. Umemura, as a result of study on lateral force distribution with the higher modes, find the triangular distribution to be insufficient especially when the period relation is such that the secondary mode is deeply excited. In certain cases it becomes necessary for the loading at the top story, as found by triangular distribution, to be increased by 20%. In ordinary buildings, foundations cannot be considered to be fixed perfectly. With foundation rotation and rocking taken into account, the period of the fundamental mode becomes somewhat longer, even with the slightest amount of foundation rotation. In the case of ground motion, the secondary mode tends to occupy a more prominent role relative to the case of a fixed foundation. The base shear is distributed best in some cases triangularly, in some cases uniformly, and in some cases in reverse directions for the lower and upper stories.

When a structure having a comparatively long period is shaken by rapidly alternating violent accelerations with a large number of frequencies, the shear

distribution would be X-type. The modification of the triangular method could be made by utilizing good average values for ground coefficient, C_g, the base shear found by the authors' proposal for the fundamental mode, C, and the base shear found for the second mode, C_2. Fig. 14 shows the development of the final line of distribution. Although not a simple operation, it does eliminate the necessity for a separate consideration of the overturning moment. It is well known by engineers that if the triangular or uniform distribution is used, the overturning moment always becomes excessive and provision for its consideration is necessary. X-type distribution reserves more shear for the upper stories than does triangular distribution. It uses the same valve of C but limits the overturning moment within a reasonable range as compared with the design shear.

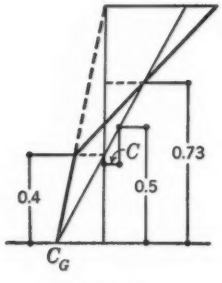

FIG. 14

Determination of Period.—The natural period of a building varies for small acceleration vibrations as well as for large acceleration vibrations. It also varies as the nature of the soil changes. The authors' proposal represents the period simply by a formula that depends entirely on the building's geometry, although the measured data vary widely. Japanese engineers have expressed some disagreement with determinining the C value solely on this basis. For instance, Mr. Naito states that the determination of the period formula based on the data contained in Fig. 8 is sound. However, if these data were assembled from the results of small acceleration vibration tests, the data should be modified for larger accelerations, such as are measured in a severe earthquake. K. Takeyama notes that the proposed formula gives results that coincide with test data obtained from small vibration tests conducted on numerous buildings by the Building Research Institute of the Construction Ministry. However, he questions the feasibility of direct adoption of this period for use with Fig. 5.

Increase of Allowable Unit Stresses.—Japanese building codes divide the allowable unit stresses into two catagories: Long-time or permanent loads (dead load and live load) and short-time or temporary loads. The allowable unit stresses for long-time loadings are increased over those for the short-time loadings by the ratio of 2 for wood and concrete, and 1.5 for steel. The proposal follows the current American practice of allowing an increase of $33\frac{1}{3}\%$ for combined loading stresses. However, the Japanese engineers are convinced that a well-balanced design will be achieved by further increasing the allowable stresses for combined loading together with the increase of the so-called load factors.

Summary.—The authors' proposal is based quite conclusively upon an overall basis period of ground motion of 0.25 sec. Seismological records in Japan indicate that the basic natural periods of ground motion have about the following values: 0.1 sec for rock, 0.3 sec for intermediate soils, and about 0.6 sec for one particular soft soil area—the Marunouchi area of Tokyo. Perhaps there is some basic difference between earthquake wave motions in the United States

and in Japan. Geologically, however, there is no sound basis for such a conclusion.

The Japanese records, indicating a basic natural period of 0.3 sec for intermediate soils, show close agreement with the 0.25 sec for the basic natural period of earthquake motion in the El Centro area. The evident similarity of these two periods suggests El Centro soil is in the class of intermediate soils. If this is true, it shows the obvious need for special consideration in the soft soil areas in the United States.

The basic natural period can be expected to vary between any two soft soil areas. It varies for any one soft soil area as the intensity of the earthquake varies. Adding to this effect the fact that a building's natural period of vibration varies with the intensity of earthquake motion complicates the problem further.

If it were possible in any soft soil area to fix exactly a basic period of ground motion that was generally applicable, the problem would be solved easily by merely modifying the spectrum for the period and magnitude of the coefficient. However, many complications must be considered, such as foundations piled to firm strata, foundation rotation, and energy dissipation.

It can be argued that, since a building on soft soil does not have the same elastic reaction to the ground motion as a building on hard soil, the energy transmitted to the building is reduced, thereby obviating the need for any special consideration. Other factors indicate a reason for not accepting this argument in its entirety. It is now accepted generally in Japan that the possibility of damage to main structural members of reinforced concrete buildings in a violent earthquake is greater for buildings on soft soils than on hard soils. D. W. Butler believes, therefore, that during some period of the violent distortional wave motion, the building, even though located on soft soils, is subjected to large accelerations and displacements. He believes this happens during the first few cycles of distortional wave motion. At this time the soil, before failure, must act elastically. The energy transmission rate to the building is at a maximum, and the energy dissipation rate from the building is at a minimum. The damping qualities of the construction have a minimum effectiveness at this time.

Not all buildings in the Marunouchi area reacted in this way to the 1923 earthquake. Some were undamaged. It is to be noted that the Nippon Kogyo Building was one of the undamaged buildings and that it was designed for an acceleration of 1/15 g. The ground acceleration was several times this value. However, the excellence of the details and construction and the location of seismic bearing walls gives this building a balance and strength that would no doubt indicate a measured capacity of resistance many times this force coefficient.

In addition to the occurrence of a longer basic natural period of ground motion, the initial tendency of the building to react with the ground motion suggests that the period and the magnitude of the C values must be increased for safe use for buildings on soft soils.

Japanese engineers are unanimous in expressing their warmest praise and admiration for a real accomplishment—the furtherance of knowledge as to

the treatment of a building so that it will better resist dynamic lateral forces. It is believed the authors' proposal is ideal for structures on hard and intermediate soils and is the most economical that can be devised. The problem of estimating the amount of reactive lateral force for important buildings on soft soils appears to be a matter in which factors other than the period of the building can be of fundamental importance.

Acknowledgment.—The Seismic Research Group of the Architectural Institute of Japan comprises the following members: M. Kobayashi and T. Taniguchi of the Tokyo Institute of Technology; K. Muto, R. Sano, K. Kanai, and H. Umemura of Tokyo University; T. Naito of Waseda University in Tokyo; S. Ban of Tokyo University; and K. Takeyama of the Ministry of Construction for Japan. Y. Otsuki serves as translator for the group and D. W. Butler serves as editor.

REUBEN W. BINDER,[12] A. M. ASCE.—This well-organized, clear, and concise paper is an outstanding contribution, covering the many phases of engineering seismology involved in the development of a lateral force code.

The authors have worked into usable form the concept presented by M. A. Biot in a paper (63*a*)[12a] on engineering seismology. Mr. Biot stated

> "Spectrum curves presented herein indicate that the effectiveness of an earthquake is inversely proportional (roughly) to the period, so that increasing the period means an increase in safety. Application of the methods developed in this paper should be helpful as a guide in extending safety rules and interpreting earthquake data."

In an earlier paper (43), Mr. Biot cautioned that

> "* * * when we possess a collection of earthquake spectrums at a given location, it is suggested that a simplified envelope should be used as a spectrum in that region. Further investigation is needed to establish such reliable spectrum envelopes and correct them for the influence of the damping and other causes of stress reduction."

The writer believes that the authors of the paper fully realize that their conclusions must be open to continued study and that they recognize the importance of obtaining considerably more statistical data. Much of the valuable information collected to date has been furnished by the Seismological Division of the United States Coast and Geodetic Survey (USCGS). Too much emphasis cannot be placed on the importance of the Survey's continued activity in this phase of the work. It can be assumed that the authors realized that arguments concerning their proposals will be centered around these factors: (1) The method used to obtain the period of the structure; (2) the character of the expected ground motion; (3) the elastic and damping properties of the structure; and (4) the duration and random character of the earthquake. Depending on the supposition made, almost any kind of a case can be built up with endless discussion resulting. Nevertheless, the materials have been pre-

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[12a] Numbers in parentheses, thus: (63*a*), refer to corresponding items in the Bibliography (see Appendix II) of the paper, and at the end of this separate.

sented in such a manner as to form a valuable contribution. It is apparent that the authors make no special claims for the proposals they have presented in the section on "Dynamic Approach to Earthquake Forces" because they state

> "The behavior of structures in earthquakes has generally been recognized as a dynamic vibration phenomenon of a transient nature. Although rigorous solutions are possible for particular ground motions applied to particular structures, these solutions are too involved and of too limited significance to be of direct value to the structural engineer. The more rigorous methods, however, should be encouraged to guide the thinking toward less rigorous but more practicable methods."

When, in addition, consideration is given to the authors' recommendations for future study and accumulation of data, it is clear that the rules that they propose must be considered as entirely empirical.

In the introduction to the paper, reference is made to the Los Angeles code, and it is stated that previous codes do not account for the dynamic characteristics that determine how structures respond to earthquake ground motion and that advances in scientific knowledge have not been reflected in current building codes. The Los Angeles City Code requires that the lateral force on buildings shall be determined by the formula,

$$F = \frac{60}{N + 4.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

in which F is the lateral force on the part and in the direction under consideration, and N is the number of stories above the story under consideration. The factor F is applied to the summation of all required loads above the story under consideration. Prior to the establishment of this formula considerable analytical and experimental work, sponsored by the Los Angeles County Department of Building and Safety and undertaken by the California Institute of Technology, was directed toward improvement of the then current design rules. The resulting report was turned over to the Los Angeles County Building and Safety Department and pointed the way to replacing the use of a constant acceleration curve with one based on equivalent acceleration, which could take into account some of the many important dynamic considerations. The formula of Eq. 13, while admittedly empirical, incorporated the use of equivalent acceleration coefficients and shear distribution based on equivalent acceleration. In adopting this formula, recognition was given to the seismic coefficient, shear envelope (effect of various modes), and the evidence of observed damage, as well as to the influence of damping and other causes of stress reduction. Since the City of Los Angeles has a height regulation limiting buildings to 150 ft, the formula did not envisage very tall buildings. The values used in this formula recognized among other things: (*a*) the importance of higher factors for the low period buildings; and (*b*) the evidence of damage to roof and penthouse structures. The writer believes that one of the important contributions of the authors is in separating the equivalent acceleration and the shear distribution into two steps.

Fig. 7 shows the shear distribution for uniform weight per unit height for various existing codes. Fig. 15 shows the effect of varying the mass at the various floor levels for one height of building and may be helpful for comparison purposes. It is interesting to note that the late Leon Nishkian, M. ASCE, in an unpublished paper presented to the Seismological Society of America in 1936, suggested that a triangular distribution be used for the lower two thirds of the height and that from that point up the shear be made constant.

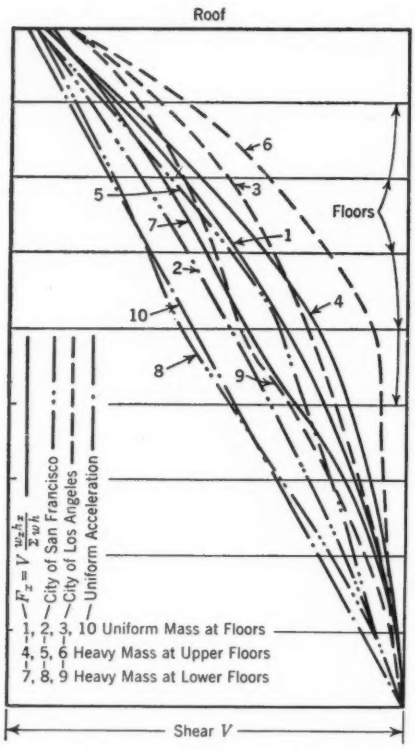
Roof
Floors
Shear V
$F_x = V \frac{w_x h_x}{\Sigma w h}$
City of San Francisco
City of Los Angeles
Uniform Acceleration
1, 2, 3, 10 Uniform Mass at Floors
4, 5, 6 Heavy Mass at Upper Floors
7, 8, 9 Heavy Mass at Lower Floors

FIG. 15.—SHEAR DISTRIBUTION CURVES

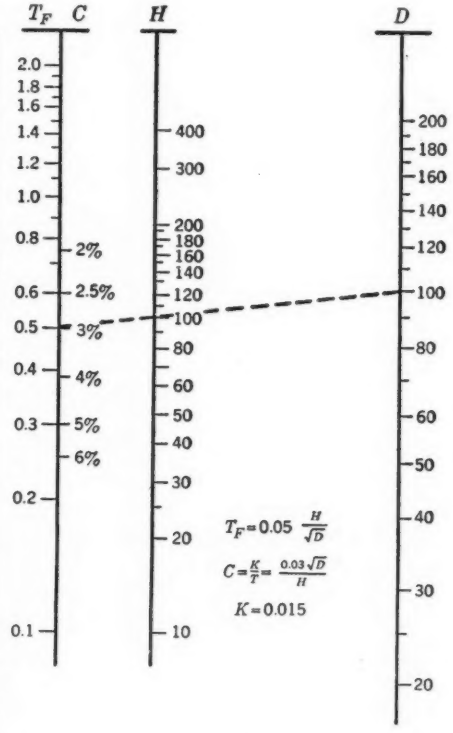

FIG. 16.—NOMOGRAPH FOR DETERMINATION OF PERIOD AND COEFFICIENT C

The free period of a structure has sometimes been considered, on a rule of thumb basis, to be from one tenth to one twentieth per second. Fig. 16 of this discussion was prepared to give rapid answers to the several formulas proposed by the authors. It may be of use for checking quickly previously made assumptions, or results obtained by calculations or experimentation with the empirical values proposed.

It is interesting to note that the authors were unanimous in their selection of $C = 0.06$ as the maximum base coefficient for buildings and of $C = 0.02$ as the minimum base coefficient. It should be noted also that the weight used (with the seismic coefficient) in obtaining the lateral force includes a percentage

of the live load. Obviously, the authors in selecting these values took into account many factors. It is assumed that one of the most important considerations was the effect of damping, on which subject much work remains to be done.

The writer considers that, in general, on the basis of the present information, the authors have taken a sound position in not establishing variable values for different types of soils.

In specifying that any resistance built into the frame (no less than 25%) may be substracted from the total lateral force requirements in order to arrive at the resistance to be carried by the other elements, the authors, in the writer's opinion, have shown considerable engineering courage and pointed the way to better structures.

For the important consideration of overturning moment the authors' recommendation is noted with interest. It has been observed by the writer that some designers, although using equivalent acceleration for the design of the structure itself, use uniform acceleration for design in connection with the foundations.

The authors' statement regarding damage to structures (in the section on "Earthquake History") deserves careful attention. It is in essence an observation that places emphasis on the importance of:

1. The provision that inertia forces should be distributed in accordance with the principles of relative rigidities; and
2. Probable high stress concentrations that may occur anywhere within a structure.

From observation of earthquake damage, important general rules may be formulated but unfortunately they cannot be quantitative. Although the observation of earthquake damage furnishes a significant yardstick, it should be remembered that with multiple variables the damage may be interpreted in many different ways.

Design rules are the result of work such as is presented in this paper and of actual observation of earthquake damage. Usually damage has been observed on structures that have not been designed for seismic forces and in which little or no provision has been made for lateral forces. It should be noted that observed damage, in general, was on box-like structures; there has been little or no experience with earthquake damage in connection with what is known as modern architecture. The importance of considering special rules for seismic design in connection with so-called modern architecture deserves considerable attention.

The authors call attention to the need of clarification for combined axial and bending stresses in columns. It should be mentioned that considerable work on this subject has been accomplished by the Column Research Council of the Engineering Foundation and it is hoped that in the near future some of the findings may be crystallized into design rules.

Since this excellent presentation might well serve as a standard reference on the subject of earthquake provisions, the writer suggests that the authors extend the bibliography given in the paper.

R. R. MARTEL,[13] M. ASCE, G. W. HOUSNER,[14] A. M. ASCE, AND J. L. ALFORD[15].—The subject of the paper is of interest to engineers in many parts of the world. The problem of earthquake-resistant design is of particular concern in the western portions of North and South America, Japan, Australia, Italy, Greece, Turkey, and portions of India and China. The paper recommends design values for lateral forces that differ from those specified in the earthquake-design provisions of existing building codes (1951), and the question arises as to whether the differences are justifiable; that is, are the recommendations presented in the paper the best earthquake-design provisions? The authors, in their paper, present some arguments, data, and analysis on which they base their recommendations, but in some instances these are faulty and invalidate the conclusions. The general problem of earthquake analysis and design is very complex, and the problem is too involved to be treated satisfactorily in a short discussion, so these remarks are limited to pointing out what appear to be shortcomings in the authors' analysis of the problem.

The conclusions of the paper are based in large part upon the so-called "standard spectrum curves," shown in Fig. 5. As the authors state, these curves actually show accelerations that are too large to be reconciled with observed earthquake damage. Therefore, one would expect that some pertinent factor has been overlooked in the preparation of the curves. The writers have recently completed a research study sponsored by the Office of Naval Research, in which some 80 spectrum curves have been calculated from 28 strong-motion seismograph records that were specially prepared by the USCGS. These curves were calculated with various amounts of damping; and typical sets of curves for horizontal components of strong earthquakes are shown in Fig. 17. It will be seen that the spectrum curves for zero damping agree with the standard spectrum curves of Fig. 5 if the sharp peaks are smoothed out. In fact, referring to the original publications it is found that these spectrum curves were calculated for zero damping, and the sharp peaks were smoothed out to obtain the so-called standard spectrum curves. Comparison of Biot's original curves with those of the writers' indicates that the former's results were subject to approximately 1% of critical damping. From Fig. 18 it can be seen that with as little as 2% of critical damping the spectrum curve is much reduced and that with 10% of critical damping the spectrum has a shape very different from that of the zero-damping curve. In a test program sponsored by the Earthquake Engineering Research Institute, the damping in a four-story, monolithic, reinforced concrete building was measured and was found to be 7.5% of critical damping (70). This probably is in the lowest range for damping in ordinary buildings, and those that are not monolithic undoubtedly have greater damping. For example, a concrete frame building with tile filler walls was found to have approximately 14% of critical damping (52*a*). With these amounts of damping, the maximum accelerations exhibited by the spectrum curve are much reduced from those given by the zero damping curve. It is indicated, therefore, that

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damping is an important factor in the response of structures to earthquakes and that the undamped standard spectrum curve used by the authors is an upper bound that should be modified when applied to ordinary buildings. The curve for the lateral force coefficient shown in Fig. 11 is based on the undamped spectrum and is not correct for structures with damping.

The authors state that there are dominating periods in the range 0.1 to 0.3 sec, but it is shown in Fig. 18 that when damping is taken into account, it cannot be said that these periods are in any sense dominant.

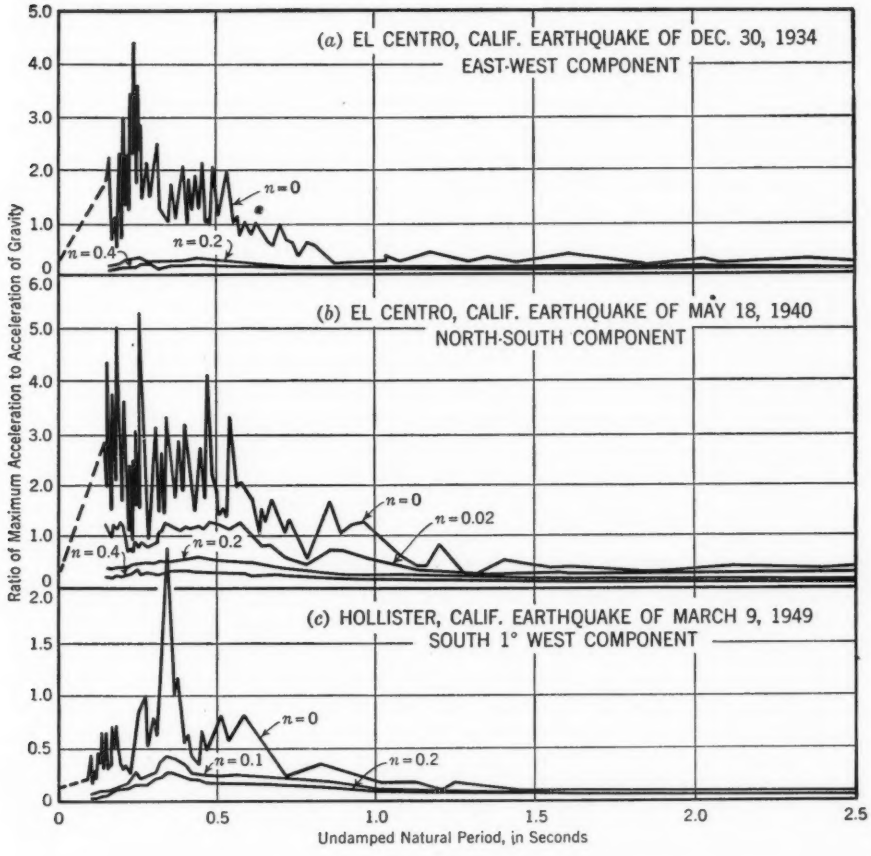

FIG. 17.—ACCELERATION SPECTRUMS

The authors assert that it is reasonable to assume the motion of the average building as 50% shear deflection and 50% bending deflection, which gives them a straight-line shear distribution. This, however, does not agree with experience. If buildings move with shear deflection, the fundamental periods of vibration are proportional to the height, whereas if the deflections are in bending, the periods vary as the square of the height. On the basis of this

criterion, experience indicates that the average building moves essentially with shear deflections, and only a minority of rather exceptional buidlings have appreciable bending deformation. This is borne out by the authors' observations on building periods, which they conclude vary as the height.

The assertion is made that only the first mode of vibration need be taken into account, and the recommendations are based upon this. It is stated that the higher modes have greater damping than the first mode and hence are quickly damped out. Experiments do not confirm this. In the aforementioned building test of a 40-ft-wide four-story, reinforced concrete building, vibrations were induced with a maximum acceleration of 8% gravity. It was found that the damping of the second mode was only slightly greater than that of the first, being 8.0% of critical for the second and 7.5% for the first. The period of the fundamental mode in these tests was 0.4 sec, and the period of the second mode was 0.2 sec. It is the opinion of the writers that modes of vibration higher than the first should not be arbitrarily disregarded. In fact, the example presented by the authors in Fig. 10, which was intended to show that the second and third modes can be disregarded, actually proves the opposite. If the first three modes are excited to the extent shown in Fig. 10, the shear at any point in the structure is the sum of the shears of each mode. During an earthquake the phase angles between the modes are not constant, so that sometimes the shears will reinforce each other, and the maximum shear will be the sum of the absolute values of the shears of the three modes. If these maximum shears are added up at each point along the height, there will be obtained a maximum shear diagram that is much different in shape from the shear of the first mode only. So, it is clear that for very small damping, which is the only case studied by the authors, the higher modes are important. It is the opinion of the writers that the question of the shear distribution and the effect of higher modes must be answered on the basis of a much more thorough analysis than is presented in the paper.

The method of estimating building periods proposed by the authors seems to be too crude for its proposed application. As shown by Fig. 8, the estimation of periods has errors as high as 100%. It does not appear proper to use a method of design that is sensitive to the period and then use such a crude method of estimating the period. For example, using Fig. 8 to obtain the period of a building in which $H^2/b = 75$, there is obtained $T = 0.4$ sec. From Fig. 11 the lateral force coefficient is 3.7% of gravity, but from Fig. 8 it is seen that actual buildings (in which $H^2/b = 75$) have periods ranging from 0.22 to 1.2 and according to Fig. 11 should be designed for lateral force coefficients ranging from 2% to 6% gravity.

The recommendation in this paper for magnitude of lateral force coefficients does not seem to be well founded. Aside from the fact that the derivation of the curve shown in Fig. 11 is faulty, as explained previously, the magnitudes of the coefficients seem too low. For example, according to this recommendation a typical 5-story building would be designed for less than 3% gravity since it has been found that typical 5-story buildings have periods of the order of 0.5 sec. Of course, the proper design values cannot be accurately determined until more experience is gained with strong earthquakes, but the concensus

of opinion as reflected, for example, in the building codes in southern California, Chile, and Japan is that the magnitude of lateral force coefficient for a typical 5-story building should be of the order of 10% gravity rather than under 3%. It is also clear that following the authors' proposals would permit flexible designs, so that buildings over four stories in height would need to be designed for only 2% gravity if desired. In fact, the proposals are practically equivalent to saying that the majority of ordinary buildings over five stories in height need be designed for wind only as regards their structural frame and that the earthquake forces can be neglected. This would be a very marked reduction from design requirements in force in regions of high seismic activity.

The authors state that the rotation of a building on its base, that is, foundation yielding, has a very considerable effect upon the period of vibration of a building and reduces the dynamic shears. It is true that if appreciable foundation yielding does occur, it will affect the vibration of a building; but to the writers' knowledge there exist no reliable data that show that, for actual buildings of average type, the amount of foundation yielding that does occur has an appreciable effect on the period or on the magnitude of shear.

It is recommended in the paper that a relatively small, slender tower on top of a building should be designed as if it were a structure resting on the ground, quite independent of the fact that it is actually supported on top of a building. It is certain that this is erroneous, for the motion of the top of a building is not at all similar to the motion of the ground during an earthquake. The actual motion of the top of a building during an earthquake is essentially composed of simple harmonic components with periods the same as the lower modes of vibration. It follows that a relatively light structure set on top of the building is subjected essentially to a harmonic forced vibration. The most important point involved is that the superimposed structure be designed so that resonance does not occur.

Because of the points brought out in the preceding comments, the writers are not in agreement with the arguments and conclusions presented in the paper. It is their opinion that the authors extended their remarks and conclusions to cover more than was justified by the amount of data and analysis that was at their disposal. It is also the opinion of the writers that all points of earthquake-resistant design that cannot be established on the basis of reliable empirical data or incontrovertible analysis should be treated with caution and that the design should be on the conservative side.

S. K. GUHA[16] AND GURDAS RAM[17].—In designing for lateral forces, such as seismic movements, codes have been developed mostly on static, rather than on dynamic considerations. This approach obscures a realistic perspective of the true situation in effect during an actual earthquake; and it never permits taking into account the various factors involved in such a rapidly changing state.

It is true that considerable field and laboratory data concerning earthquakes have been assembled in the past century; but there is still an acute need for

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studying the types of complex ground motions associated with destructive shocks. The dynamical theory ordinarily developed on the basis of sustained and simple harmonic motion of the ground is not considered sufficient to take into account all the hazards that arise during an earthquake. Figs. 3 and 4 show the extreme complexity of the ground motion near the destructive zones, and it is realized that, at greater distances from the epicentral region, these complexities would be simplified as the very high frequency oscillations become dampened and the remaining waves become lengthened. A structure can naturally withstand a smooth and sustained vibration better than it can withstand the complex nonsystematic oscillations depicted in Figs. 3 and 4.

It is also necessary to know the prevalent seismic spectrum of a zone—depending mostly on the geology of the region. Such a spectrum, for California, was prepared by Beno Gutenberg, et al (52), from analyses of a large number of seismograms of different observatories. In design, this regional influence on the applied, actual seismic spectrum should also be incorporated because different regions may react to produce widely different seismic spectrums.

The seismic damage to structures described in this paper has not been corrected to the prevailing geology of the region, although the importance of such correction has been emphasized, in a general way. It is quite possible that a consolidated foundation would reduce the seismic intensity, and consequently the damage. The detailed isoseismal study of the 1906 earthquake in California revealed this fact clearly.

Syun'itira Omote (71) and others have studied the effect of geology on damage rate in great detail and have concluded that the depth of overburden has a predominant effect. In the case of soil, they found that the damage rate increases with depth of alluvium, reaching a critical value at 35 m (115 ft). Similar conclusions were also reached in the case of mud with a critical depth of about 6 m (20 ft). It is possible that the existence of critical values during seismic movements signifies the presence of some kind of resonant vibration in the overburden layer (at a critical depth). Although Mr. Omote's study was limited to wooden structures, the general conclusions apply to other types of structures.

There has been a distinct difference in the damage sustained by structures with and without frames, especially steel frames. Possibly, such differences are due to the dynamic characteristics of the framing. It is possible that the dynamic characteristics of the filler walls, masonry or concrete, are obscured by those of the framing during transit vibrations, with the frame responding much more quickly than the filler wall material. Of course, a complete dynamic analysis can be made only when the elastic properties and the bond between the filler walls and the frame is known. These factors determine the dynamic behavior and the potential damage to a structure during a transient vibration. The difference in damage with and without the frame can be systematized or assessed only when the foregoing properties of building frames are known.

It is also possible that the filler walls in a framed structure act simply as loads on the framing during vibration. This assumption ignores the existence of the wall material even as a weakly participating element working

with the frame. The excellent record of framed structures during earthquakes, both in the United States and in Japan, argues for the tenability of this concept especially for the transient impulses of an earthquake. However, for sustained and harmonic vibration, it is likely that the filler walls may actually contribute something to the resistance of a structure.

During an earthquake, a masonry or concrete structure with a steel frame is expected to respond in a manner completely different from that of a structure without framing. The slow reaction of masonry material in a framed structure is due to the quick transmission and response of the elastic energy by the framing material, especially steel. This difference in behavior becomes more acute for higher structures.

In general design analysis, the base of the building is assumed to be clamped; but Kiyoshi Kanai observed (72) that they act as clamped bases in the relatively longer vibration period and as hinged bases in the shorter periods. Thus, the foregoing dynamical picture envisaged for a building may need to be changed, considering the applied frequency of the seismic waves. In the hinged condition, the reception and transmission of the waves will not be similar to that prevailing in a clamped state. The authors have not considered, in sufficient detail, the effect on the dynamic design of diverse seismic spectrums.

In any appraisal of seismic damage, it is suggested that proper attention be given to the direction of ground motion with respect to the structure. A given seismic spectrum may actuate the structure in one direction whereas it may fail to do so rationally in the other direction, depending on the natural frequencies in the respective directions, and on the degree of interaction between the two. In an asymmetric system, rotation and translation are each induced by a translatory ground motion. The rotation is caused by the interaction of a structure in its several planes, caused by asymmetry. Generally, the motion along one axis is accompanied by motions along the remaining two co-ordinate axes. Thus, the direction of the ground motion with respect to the structure plays a dominant part in its response. Of course, for design purposes, one must apply standards for determining the maximum response in a given structure under the seismicity that is prevalent in the region.

The distortions produced in a structure, under the foregoing conditions, are influenced by both flexural and shear modes of vibration; the predominance of one or the other is determined by the height-width ratio (H/b) of the structure. It is generally believed (and recognized theoretically) that distortions are caused mostly by flexural modes for higher values of H/b and by shear modes for lower values. For values of H/b greater than 4, distortions can be assumed to be mostly flexural. The predominance of the one mode or the other, for various values of H/b, will defeat all efforts to systematize the natural periods of buildings as a whole. Fig. 8 suggests that the authors gave preference to the flexural mode of vibration, which is directly related with H^2/b. For complete shear distortion, the period will be directly proportional to H. The final correlation that the authors report for the natural period, and H and b, suggests that data taken as a whole would not show either flexural or shear distortion but a combination of both. This situation could probably be avoided if T was found as a function of H^2/b for buildings having H/b-ratios

greater than 4 and, as a function of H for buildings having H/b-ratios less than 4. Of course, the value 4 has been selected rather arbitrarily. It is realized that there is a zone for H/b that will overlap both T_1 and T_2 to a certain extent because, under the circumstances, there cannot be any sharp division between shear and flexural distortion in structures, in terms of the ratio H/b.

The vibrational characteristics of a structure will certainly be affected by the elastic yielding of the foundation. The resultant effect will be an increase in the deflection of the structure and, consequently, an increase in the period. The yielding affects the fundamental periods of translation as well as those of torsion very appreciably. The effects on the overtones are also felt in this respect. It should be an inherent part of dynamic design procedure to take into consideration the relative elastic behavior of foundation and structure.

From a general dynamic viewpoint, it would seem basic that the fundamental mode should maintain a comfortable envelope for higher harmonics in respect to shear distribution. Although the authors have called attention to increases in shear in higher parts of structures, for higher harmonics, it is recognized that the fundamental mode may still have a good envelope for higher harmonics, in dynamic design. The absolute value of impressed shear in those parts is not considerable. This has been demonstrated by statistical analyses of the usual damage to buildings of different heights; and it has been found that, in buildings, serious damage occurs generally below the mid-height of the structure. This confirms the observation that the fundamental mode is of predominant importance in the dynamic approach because, as such, extreme precautions may not be necessary in higher parts. Sometimes, of course, it is found that higher parts of a structure suffer from small cracks, which may be the result of elastic nonhomogeneity in the structure producing differential movement near the region. Although model experiments show increases in shear for third harmonics and second harmonics at higher levels, it is felt that this is an exaggerated effect characteristic of the model, and may not have much bearing in the prototype.

Model experiments and theory sometimes seem to indicate that a structure will react with very high dynamic shear and flexural stress values, especially when the known seismic spectrum has prevalent frequencies approximating the natural frequency of the structure; but the unexpectedly excellent record of moderate structures, in seismic zones such as Japan, has given rise to a belief that damping due to internal friction in the structure, and to dissipation of energy in the elastic foundation, will prevent the structures from attaining large vibrational energy and thus will help in reducing dynamic shears and vibrational stress. Although the authors have recognized the dynamic influence of the damping process, they have not shown, clearly, a practical method of incorporating this factor in design to avoid great expense as compared with the cost of other shock-proof measures.

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M. ASCE; HENRY L. MARCHAND;[23] HENRY C. POWERS;[24] JOHN E. RINNE,[25] M. ASCE; GEORGE A. SEDGWICK,[26] A. M. ASCE; AND HAROLD O. SJOBERG,[27] A. M. ASCE.—The Joint Committee appreciates the study which the discussers have made on the controversial subject of earthquakes. The controversial aspects arise from the two basic sources: (1) Incomplete understanding of earthquake phenomena; and (2) criticism of simplifications which lack complete rigor.

The first source is being resolved by continuing research and the Joint Committee has indicated several specific problems needing further study. Included was the subject of damped spectra for which Messrs. Martel, Housner, and Alford are developing some data. Need for world-wide recording of ground motions in moderate and strong earthquakes, on an accurate and uniform basis, was also acknowledged. Research along these lines should be pursued more intensively.

The second source comes from the fact that, even under idealized conditions, earthquake phenomena are very complex. Completely rigorous analyses are not warranted in the day-to-day application to structural designs. Simplifications are necessary and it must be recognized that when these are made there is room, and perhaps justifiable room, for criticism because simple rules raise exceptions and complex rules become overbearing.

The discussion by Messrs. Martel, Housner, and Alford requires special comment. When some important aspects of the paper are properly understood, the arguments presented in this discussion do not "invalidate the conclusions." The analysis of some eighty spectra, completed after the April, 1951, publication of the paper, is a fine contribution. However, these spectra can be shown to justify rather than refute the recommendations in the paper. It must also be recognized that where "reliable empirical data" and "incontrovertible analysis" are lacking, judgment backed by available facts and engineering experience must be relied upon to meet the demands for building designs which cannot wait for such data. A purely academic approach is impractical and probably will be for some time to come. Future studies should either corroborate and reinforce the recommendations or indicate that modifications are required to keep engineering practice in step with scientific developments. Until considerable new factual data are reported, however, it is necessary to apply engineering judgment to the data at hand.

The discussions have not revealed any aspects of earthquake design forces which had not been considered carefully by the Joint Committee. Existing earthquake codes are unrealistic in some important respects and too severe to be justified as minimum requirements of law. In fact, certain code requirements may be more harmful than beneficial.

The following comments relate to specific aspects of the earthquake problem as covered in the discussions.

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[25] Civ. and Structural Engr., Standard Oil Co. of California, San Francisco, Calif.
[26] Structural Engr., W. P. Day, Archts. and Engrs., San Francisco, Calif.
[27] N. H. Sjoberg & Son, Gen. Constrs., Oakland, Calif.

Ground Motion.—The Japanese discussions emphasize variations in "natural periods" of the ground. The conclusion is that the peak of the spectra should shift to the right, going from firmer to weaker foundation materials possibly with peaks at 0.1 sec for rock, 0.2 sec for intermediate soils, and as high as 0.6 sec for soft soils. Spectra available to the Joint Committee to date do not exhibit this much variation.

The Joint Committee is not familiar with the details of the instruments currently used in Japan for measuring ground motion, and more specifically, ground accelerations needed to determine the earthquake spectra. Assuming these instruments are comparable to those used by the USCGS, records of acceleration should be obtained for different soil conditions. Spectra can then be determined using an electric analog such as that at the California Institute of Technology, in Pasadena, or by other analytical or experimental methods.

Messrs. Guha and Ram state that a structure can withstand a smooth sustained ground motion of a distant earthquake better than the complex nonsystematic oscillations in the epicentral area. There are two conflicting considerations here, namely, the higher intensities near the epicenter and the greater possibilities for resonance effects under the influence of the smoothed, but less intense ground motion at greater distances. The usual experience has been that the greatest damage is inflicted in the epicentral area, indicating that intensity normally governs, but resonance effects at some distance might be damaging to particular structures.

Spectra.—The development of damped spectra by Messrs. Martel, Housner, and Alford is a valuable contribution. Mr. Binder and the Japanese investigators were also aware of the need for such data. Messrs. Guha and Ram emphasize the need for spectra in particular zones which may vary depending especially upon the geology of the region.

Mr. Martel's complete test data have been made available to the Joint Committee and the following observations are the result of an engineering review of all these data:

(1) The undamped and damped spectra were obtained using an electric analogy. This is an excellent method for quickly and conveniently determining spectra from ground-acceleration records obtained with the modern strong-motion seismograph. It is strongly recommended that such spectra be determined promptly for all future records of ground motion.

(2) It is unfortunate that more acceleration records of major earthquakes are not avilable. For a modified Mercalli scale intensity of X, the El Centro record of May 18, 1940, is the only acceleration record extant. It is questionable whether spectra determined from records of minor intensity can be interpreted to represent conditions in a major earthquake. This is especially significant for damped spectra.

(3) Damping spectacularly reduces the spectral values of all of the earthquake motions investigated, especially in the region of short periods. The ratio of maximum value (at short periods) to the minimum value (at long periods) is also reduced.

(4) With damping of 0.10, the maximum spectral value of the El Centro earthquake of 1940 is of the order of 1.0 g, indicating that the idealized damping analysis still does not reduce the quantitative spectral values to coefficients practical or necessary for design. Therefore, the damped spectra like the undamped spectra, must be considered as qualitative curves to which practical design coefficients must be assigned by engineering judgment. This will be influenced by observations of earthquake damage and structures undamaged, the allowable working stresses, and other design considerations which the practicing structural engineer is most competent to evaluate.

Periods of Structures.—The method for establishing design earthquake forces on structures proposed by the Joint Committee requires the predetermination of the fundamental period of the structure. The factors entering into the period are complex, and a rigorous determination does not appear warranted generally. The formula involving only the geometry of the building was intentionally selected to give periods usually on the low and conservative side. Further study should be made to establish a better formula. Until this is done, there is little to be gained by making a nominal distinction suggested by Messrs. Guha and Ram of relating the period to the height H for low buildings and to H^2/b for taller buildings. A more exact formula will involve at least some of the following factors—soil conditions; elastic properties of the materials of construction, dimensions of the structural elements; number, size, and spacing of openings; number and construction of partitions; and mass and its distribution.

The present Los Angeles, San Francisco, and the Uniform Building Code of the Pacific Coast Building Officials Conference (PCBOC) specify coefficients varying only with the number of stories of a building, a procedure justified by dynamic considerations which basically involve the period. The introduction of the period as the criterion for the lateral force coefficients acknowledges the importance of the proportions of a building and is a forward if not exact step.

Messrs. Martel, Housner, and Alford state that

> "* * * there exists no reliable data that show that, for actual buildings of average type, the amount of foundation yielding that does occur has an appreciable effect on the period or on the magnitude of shear."

This is contrary to many expressed views, as well as the known behavior of structures (22) (59) (63) (68).

The shortcomings in the method of predetermining the period does not invalidate the general approach. It would appear logical and practical to introduce the criterion of relating the lateral forces of earthquake to period, and then set about to develop more reasonable means for predetermining the period—reasonable both as to accuracy and workability. The nomograph solution of the equation for period and the corresponding coefficients as presented by Mr. Binder, are useful tools pending the development of a more accurate formula.

Distribution of Base Shear: Effects of Higher Modes.—Mr. Martel suggests that the triangular distribution of the base shear arbitrarily disregards modes higher

than the first. Such is not the case. The point that is made by the writers is that the shape of the fundamental mode shear curve—that is, the shear curve corresponding to the triangular distribution of the base shear—is a reasonable envelope to contain the higher mode shears. The Joint Committee is well aware that the actual response of a structure, especially a tall multi-mass structure, is a composite of the fundamental and higher modes. Under the most adverse conditions, these modal responses add to produce a shear diagram with an hour-glass shape; but, from a practical view, present knowledge does not warrant the design of a multi-storied building with other than an increasing shear resistance from the top to the bottom of the building. The inherent increasing lateral resistance built into a structure to take care of the increasing vertical load makes this both practicable and desirable. Furthermore, this accords with the observations of Messrs. Guha and Ram and others that, almost without exception, building damage in earthquakes has been in the lower half. It is the Joint Committee's contention that the triangular distribution of the base shear will provide sound structures for higher modes as well as for the fundamental.

In arriving at the triangular distribution, the case of a building deflecting half in shear and half in moment was assumed. This was not represented to be an average building. In fact, it was acknowledged to be a slender building of the Alexander Building type in San Francisco. Most buildings are not so slender and would deflect more in shear than in moment. Therefore, the triangular distribution is conservative with respect to the fundamental mode and provides some excess resistance in the upper parts of a multi-storied building to take care of the second and higher modes which have secondary maximum values of shear in these parts.

Messrs. Muto and Umemura suggest combining the shear curves in the fundamental and second modes, resulting in the aforementioned design shear curve with the hour-glass shape. An advantage in this is that the arbitrary restriction on overturning moment considerations need not then be specified. The moments would automatically decrease near the base, compared to the values computed from the triangular distribution of the lateral forces. As new research is undertaken, this approach may have distinct possibilities. For the present, the writers prefer to design for increasing shear from the top to the bottom of the building and to introduce a limitation on overturning considerations.

Coefficients.—The Japanese group is strongly inclined toward different sets of coefficients for different soil conditions, with higher factors for weaker soils. Although some codes have this feature, most engineers feel that this refinement is not justified for California conditions. The Uniform Building Code of PCBOC, for example, varies the coefficients with the seismicity of the region but not with the soil conditions. There are conflicting effects of soil conditions on the coefficients and until more data are found to establish how these are interrelated, a single set of coefficients is specified.

The actual values assigned to the design coefficients depend upon a number of factors including the seismicity of the area, the definition of the loads to which the coefficients are to apply (percentage of design live load), the

allowable increase in working stresses, the earthquake damage experience, the review of structures which have successfully withstood major earthquake, and the basic consideration of what these design coefficients are to accomplish, assuming they are properly applied.

The Joint Committee disagrees with Mr. Martel's comments on the coefficients. Japanese conditions cannot be compared directly with California conditions. In Japan, the seismicity is higher, the allowable working stresses are higher, and it is understood that arbitrary allowances are made for non-computed elements of the building which are recognized to add to its lateral resistance.

Even for Japanese conditions, the writers understand that it was the practice of Mr. Naito to design buildings for lateral coefficients of one fifteenth of gravity ($C = 0.067$) previous to the 1923 Tokyo Earthquake. Little damage was experienced in these buildings, the Nippon Kogyo Bank, for example, being virtually undamaged although surrounded by almost complete destruction of other buildings.

In California in areas now specifying the Los Angeles formula for the shear coefficients, these coefficients are not 0.10 for a five-story building, but 0.071 for Zone 3 areas and 0.036 for Zone 2 areas. These coefficients are applied to dead load only. For comparison,the Joint Committee's coefficients on a typical five-story building are given in Table 3, and are 0.0468 in the long direction of the building, and 0.0331 in the short direction, applied to dead load plus 25% of live load (or 50% of live load for storage buildings). Thus, the coefficients recommended are both somewhat lower and somewhat higher than those now applicable in Southern California. It is true that the code recommended in the paper recognizes flexibility by providing different coefficients in the narrow and long direction of the building, but this has a rational basis as the periods in the two directions are not normally equal. It is very doubtful that the coefficient would ever be as low as 0.02 for a four-story building. By the formula for period given in the paper, this would require a building 7.1 ft to 12 ft wide, corresponding to heights of 40 ft to 52 ft which is the range that might be expected in a four-story building.

The Joint Committee is satisfied that the proposed coefficients and code provisions are adequate and reasonable minimum legal requirements for California conditions. The establishment of these coefficients is not an academic problem but requires the judgment of those members of the profession who are regularly engaged in the practice of structural engineering.

Allowable Stresses.—The Joint Committee was guided with some reluctance by the prevailing practice in the United States of using allowable increase in stress of one third for abnormal combinations of vertical and lateral load. A better balanced design is obtained by increases in stress and higher coefficients—the practice followed in Japan. These increases have been recommended for further study.

Small Towers on Buildings.—The comment by Messrs. Martel, Housner, and Alford, that small towers on top of buildings are subjected essentially to a forced harmonic vibration of periods corresponding to the lower modes of the building below, is correct. The Joint Committee would prefer to specify

higher coefficients for the towers rather than introduce an elusive statement about avoiding periods in the tower which might be in resonance with the periods of the several modes of the building below.

It must be emphasized that present codes make no provisions for towers or how they should be handled. Under some codes, the towers may even be treated as additional stories and so reduce the design lateral load on the entire building, erroneous though this procedure is.

Corrections for Transactions.—In Eq. 8 change W to w; in Col. (10), Table 3, the computation for C_1, delete "0.9468–"; and in references (22) and (23) change "Bachelor of Science" to "Engineer."

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